Mat-Supported Jack-Up Foundation On Soft Clay – Overturning Storm Stability

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Abstract

The Maleo Producer is a converted Bethlehem JU 250 (1970's design) mat-supported jack-up that is now operating as a gas production platform in Indonesia. The platform is owned and operated by Global Production Solutions (GPS) and produces gas for Santos. Extensive investigations into the sea bed soils around the in-place structure and analyses of its foundation static and dynamic strength characteristics were undertaken in the first half of 2007. This paper provides an overview of the calculation methodology of the structure's overturning resistance site to storm loading on the soft clay at the site. The soil strength characteristics and method of computing bearing capacity and overturning resistance of the mat foundation are described. The methods adopted were accepted by ABS for class approval of the structure as an offshore installation.

Shallow Foundation Capacity

Mat supported jack-up foundations on soft clays may be assessed using the shallow foundation capacity equations given, for example, in API RP2A. However, while these equations deal with bearing capacity for shallow mats and strip footings, the designer is left with uncertainties as to how to treat mats with large cut outs and fingers around the mat slot. The uncertainties become greater when the overturning resistance is considered. Part of the dilemma is to assess when a mat with cut outs should be treated as an equivalent area mat and when it should be treated as a series of strip footings. In most soft clay offshore sites the difficulties of assessing mat foundation capacities are further compounded by the increasing strength of the soil with depth.

In very soft soils, on level sea beds, mat rigs will penetrate into the sea bed until a depth is reached where the soil bearing capacity is just sufficient to support the weight of the structure (less it buoyancy) if the mat is kept level and is caused to penetrate slowly. The consequence of the structure not penetrating evenly, but rocking during penetration may result in deeper final penetration of the mat. The effects of uneven penetration causing the structure to tilt back and forth and the bearing pressures to increase and decrease from one side of the mat to the other are difficult to assess. Such effects are considered to have played a significant role in the installation of the Maleo producer.

Jack-up drilling rigs are often not pre-loaded as is the almost universal practice with independent leg jack-ups. However, pre-loading mat rigs can be of critical importance to their storm survival in soft clays.

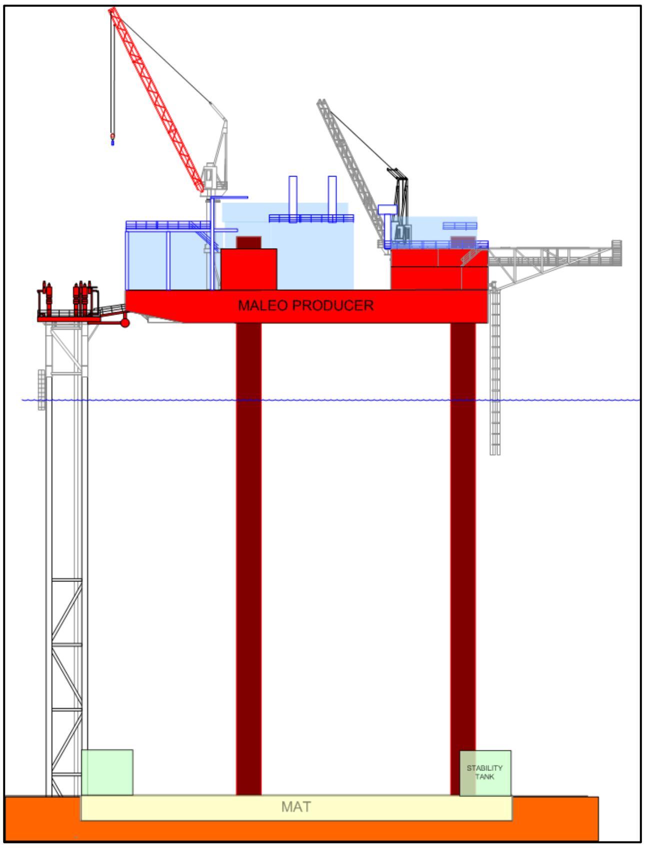

FIGURE 1 – Maleo Producer Side Elevation

The Maleo Producer

The Maleo Producer is connected to a lightweight wellhead platform with a hinged structural connection and with flexible flow line jumpers to the wellheads as shown in Figure 1.

Three legs, each 12.0 feet (3.66 m) outside diameter and with wall thicknesses varying from 3" to 1.5" (75mm to 38mm) support the hull, or deck, gas production equipment and accommodation facilities on a mat sitting on the sea bed. The mat is a steel box structure, 10 feet (3.05 m) thick with multiple internal compartments. The water depth is around 187 ft (57 m).

The Maleo Mat

The mat plan view is shown in Figure 2. It has 2 feet (0.61 m) deep skirts around all edges, including the cut out and slot areas.

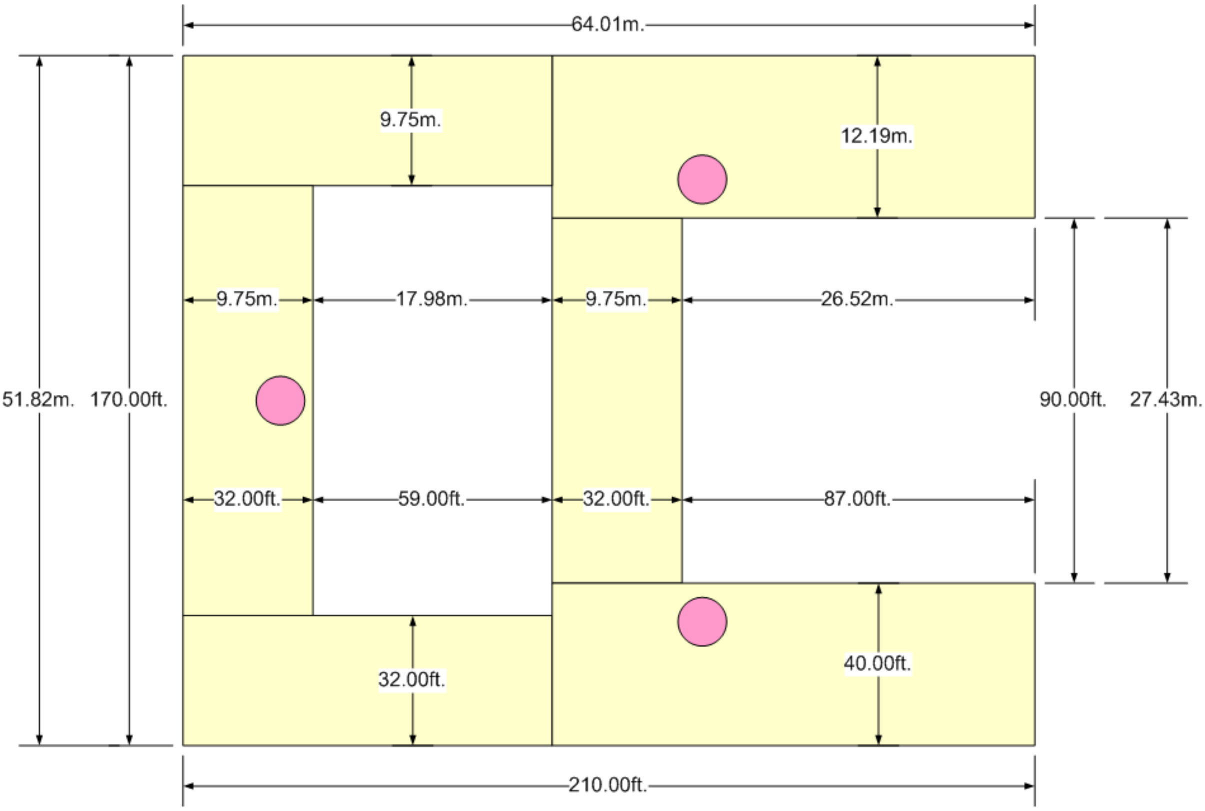

FIGURE 2 – Mat Dimensions and Leg Locations

The mat bearing area (excluding cut-outs) is 21616 sqft (2008 sqm). The rectangular mat area, including cut outs is 35700 sqft (3317 sqm). The ratio of these areas is 1:1.65.

Soil at the Site Before Mat Placement

The soil is a normally consolidated clay. From a site investigation by Fugro before the structure was placed on location the soil shear strength was determined to be characterized by an undrained shear strength of 40 psf (1.92 Pa) at the sea bed surface, increasing linearly with a strength of 7.83 psf/ft (1.23 kPa/m). The submerged weight of the soil was predicted to be 25 lb/cuft (400 kg/cum) at the surface, increasing linearly to 34 lb/cuft (545 kg/cum) at a depth of 39 ft (12m).

Predicted Mat Penetration Into The Sea Bed

Initially using the concept of a foundation placed in an equilibrium position, or "wished in place" (with the soil that would be displaced by the mat somehow having been removed) a depth at which the bearing capacity of the soil would be sufficient to support the mat can be predicted.

The maximum load imposed on the soil by the structure (its buoyant weight) during preload was 12970 kips (5883 tonnes). This results in an average bearing pressure over the mat area of 600 psf (28.7 kPa).

Treating the mat as a series of infinitely long strips, the average width of each strip has an average width of 35.09 ft (10.70 m). This width is determined from taking the lengths of the two sides and the lengths of each transverse strip shown in Figure 2, and dividing the area by this length.

Neglecting the buoyancy term from the overburden pressure, the bearing capacity at a depth z was computed from:

q_ult = F.(Nc.Cu_base + kappa.B/4).Kc

Where:

F	=	Bearing capacity correction factor (soil strength increasing with depth)
Nc	=	Bearing capacity factor = 2 + pi = 5.14
Cu_base	=	undrained shear strength beneath base of foundation
kappa	=	undrained shear strength increase with depth (7.83 psf/ft, or 1.23 kPa/m)
B	=	average width of strip (35.09 ft, or 10.7 m)
Kc	=	depth and shape correction factor

The term F can be computed from Figure A.1 in ISO 19901 Part 4 2003, as a function of (kappa.B)/Cu_base.

Foundation Base Issues – Skirt Effects

Figure 3 shows a cross section through the mat partially penetrated into the sea bed.

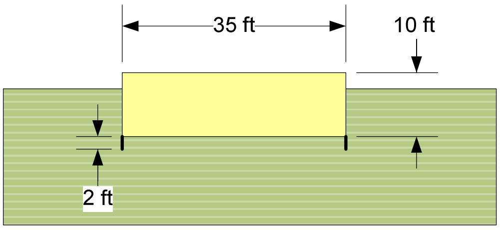

FIGURE 3 –Mat Strip Cross Section

The base of the foundation might be taken as the bottom of the skirts or as the bottom of the mat. The skirts certainly have the effect of trapping some soil beneath the mat. The presence of the skirts make it more reasonable to treat the base as being rough rather than smooth when determining F, using Figure A.1 in ISO 19901 (taken from Davis and Booker, Reference 1).

Initial Mat Penetration Predictions

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.996	kappa1.Baverage/cu.0	4.285	4.285
Nc.Kc	5.12	shape factor for pure vertical on kappa1 soil, scv	-0.053	-0.053
Depth Mat Bot.	3.08 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0036	-0.0036
Skirt Depth	0.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.026	0.026
Av. q_ult	600 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.996	0.996

TABLE 1 – Iterated Solution for Equilibrium Depth – Skirt Depth Ignored (Original Soil Strength)

Table 1 shows a solution of 3.08 feet for the equilibrium mat penetration depth into the soil with the shear strength profile predicted to exist before the mat was set in place. The skirt depth has been set to zero in this case, and the overburden pressure has been made zero by setting the soil submerged weight to zero, and the cohesion (effective friction) on the mat and external skit faces has been set to zero.

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.995	kappa1.Baverage/cu.0	4.285	5.670
Nc.Kc	5.12	shape factor for pure vertical on kappa1 soil, scv	-0.053	-0.070
Depth Mat Bot.	1.08 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0036	-0.0048
Skirt Depth	2.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.026	0.009
Av. q_ult	600 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.996	0.995

TABLE 2 – Iterated Solution for Equilibrium Depth – Skirt Full Depth Used (Original Soil Strength)

Table 2 shows predictions for mat penetration if the foundation bottom is taken at the bottom of the skirt tips.

Installation and Pre-Loading

The structure was installed in July 2006. Pre-loading was effected using tanks and void spaces in the hull. Initially during installation the mat was set down level on the level sea bed within the target distance tolerances from the existing wellhead structure.

As the hull was jacked up relative to the mat, the center of weight on the soil beneath the mat shifted longitudinally owing to the changing position of the longitudinal center of the hull buoyancy. The eccentric loading on the mat resulted in the structure tilting down towards the bow. The tilt was estimated to have been about 2.5º and occurred in a period of about 10 seconds before the hull was out of the water. This would have caused the bow of the mat to penetrate about 4.5 feet (1.4 m) deeper than the mat average depth and the stern to be 4.5 feet higher than the mat average depth.

The tilt down towards the bow was over-corrected and a tilt down of about 0.8º towards the stern gradually occurred over a period of about 45 minutes. The hull was finally brought to maximum pre-load condition after around two days and remains now (one year later) at an angle of approximately 0.4º down by the stern and 0.3º down to port.

The final average penetration of the mat was estimated to be 8.75 feet (2.67 m) to the bottom of the mat plate, with the skirt tips being 2 feet deeper. This estimate was made based on the existing wellhead structure's elevation.

Local Site Investigation Following Installation

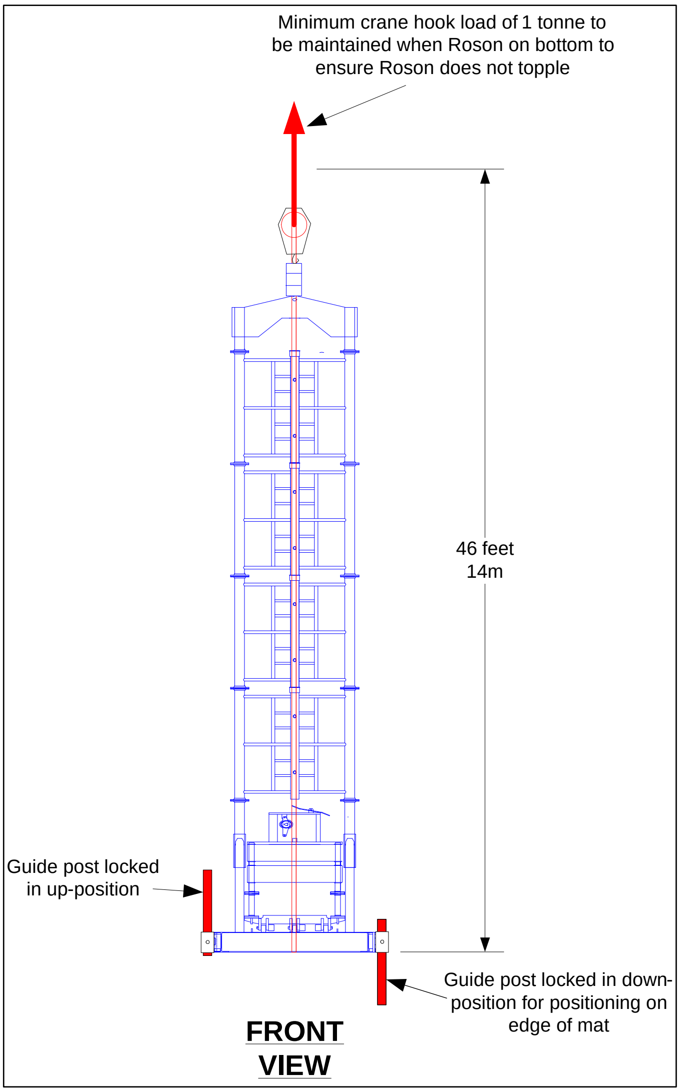

FIGURE 4 – Roson Detail Showing Push Rod String in Center

A comprehensive local site investigation of the soil in the mat effected zone was undertaken in April 2007, some nine months after the structure had been installed. This local SI was undertaken with a Roson machine deployed from the deck of the Maleo Producer using the structure's own cranes and some purpose-rigged lines and winches. The Roson machine, supplied and operated by Fugro, was modified slightly enabling it to be accurately positioned on the top edge of the mat and to push rods with cones, T-bars and a shear vane to a depth of 40 ft (12 m) from the mat top plate. Additional Roson "boreholes" were made away from the mat to the limit of the crane's reach at several locations on the port side and at two locations on the starboard side.

Figures 4 and 5 show the Roson with its central string of rods on the bottom of which is attached the instrument.

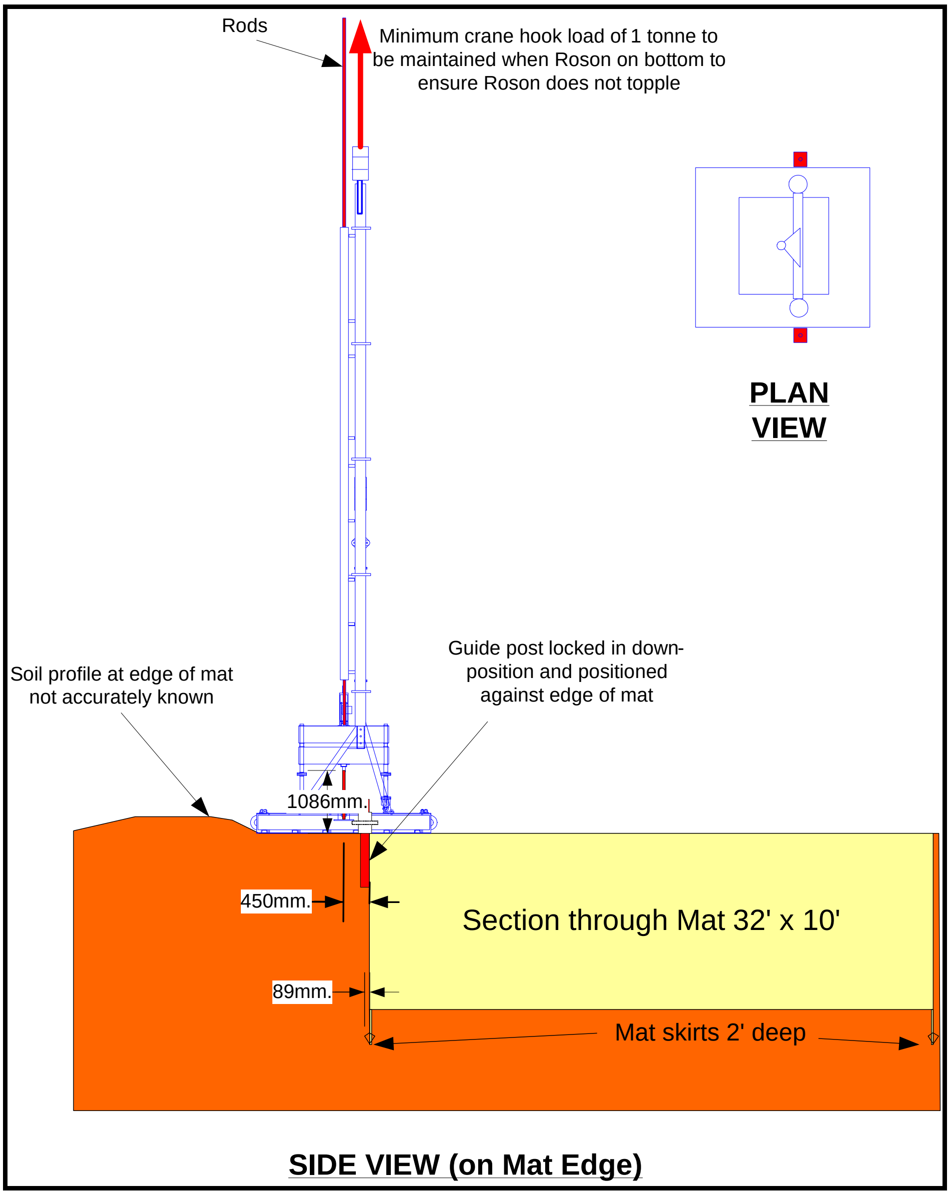

FIGURE 5 – Roson Positioned on Top Edge of Mat

Figure 6 shows the borehole locations. BH4 was particularly challenging, being beneath the hull of the Maleo Producer.

Figure 7 shows soil displaced above original sea bed with displacements derived from the soil "boring" data.

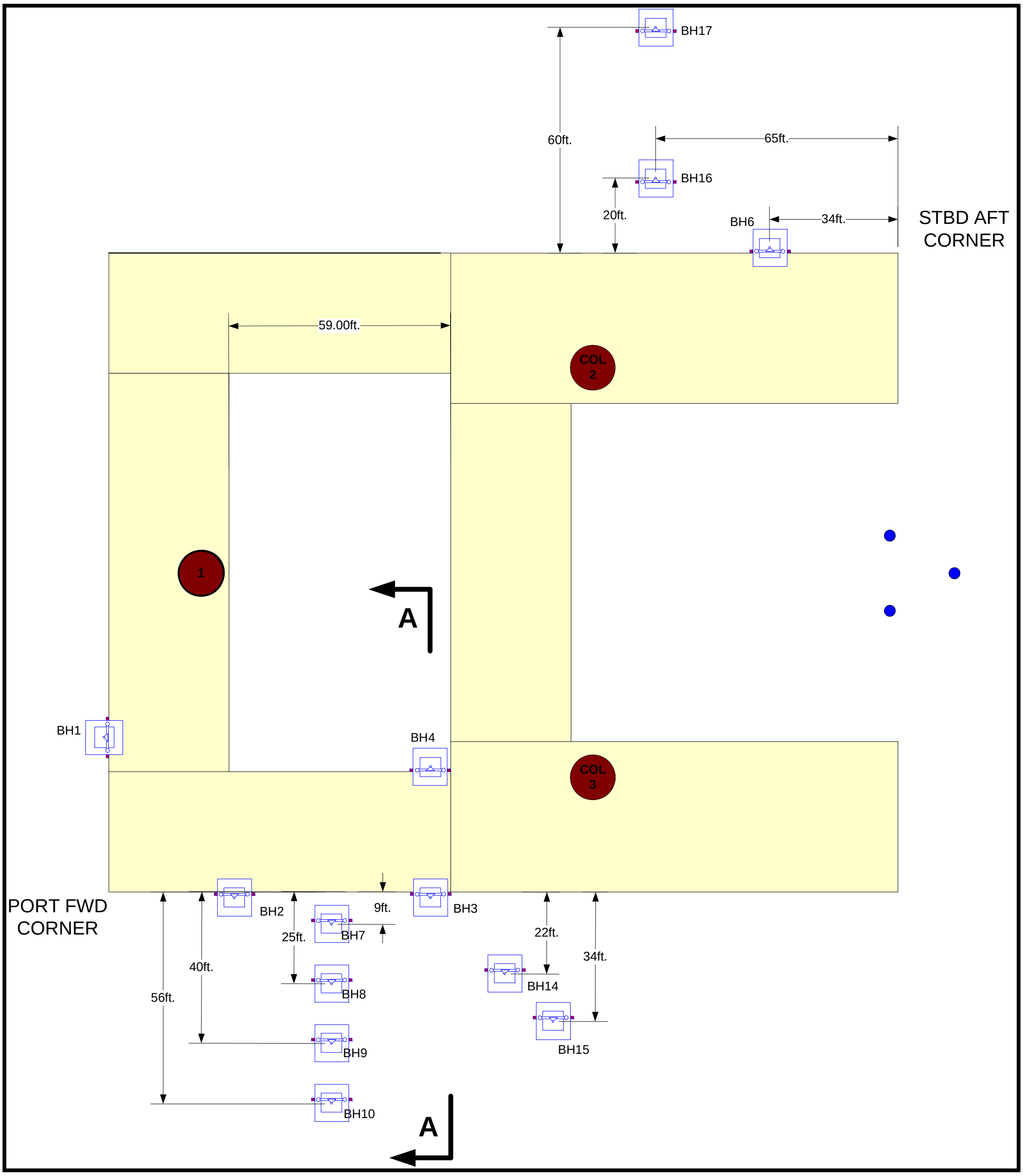

FIGURE 6 – Borehole Locations (Roson Machine Locations)

From the soil "boring" logs it was noted (special thanks to Jean Audibert) that a sharp spike was observed to occur in both the CPT and T-bar data and was believed to represent a very useful marker stratum that was deep enough not to have been affected by the soil wedge (Prandtl zones) that would have formed during penetration of the mat into the seafloor. By using this marker stratum as a common elevation datum, it became possible to estimate the thickness of the soil mound by adjusting the reference depth of each CPT and T-bar sounding to match the location of the spike on each data profile

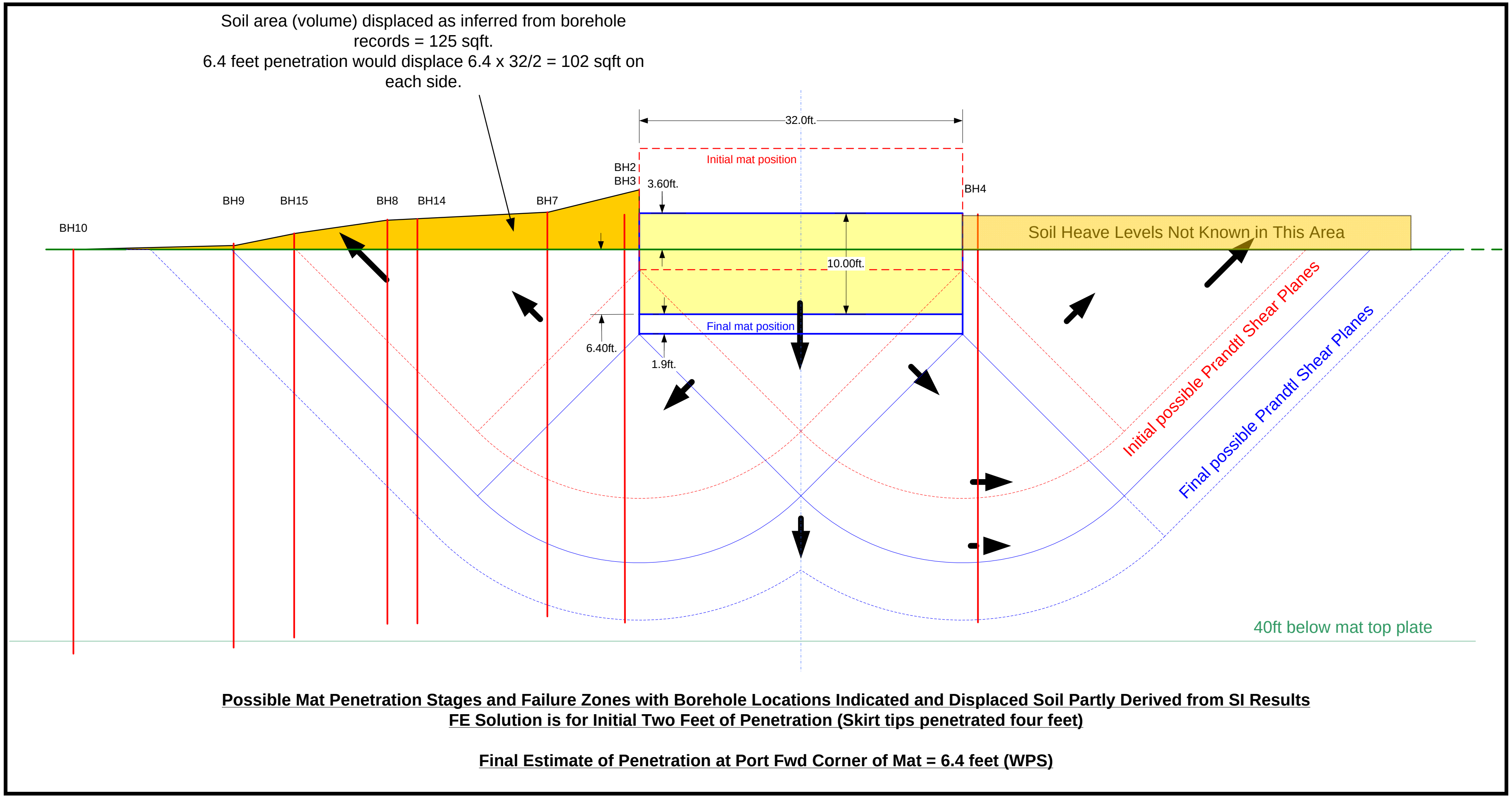

FIGURE 7

Figure 7 shows the relative positions of the boreholes made with the Roson around the port forward quadrant of the mat (see also Figure 6). The classical Prandtl geometries and direction arrows have been added by the author.

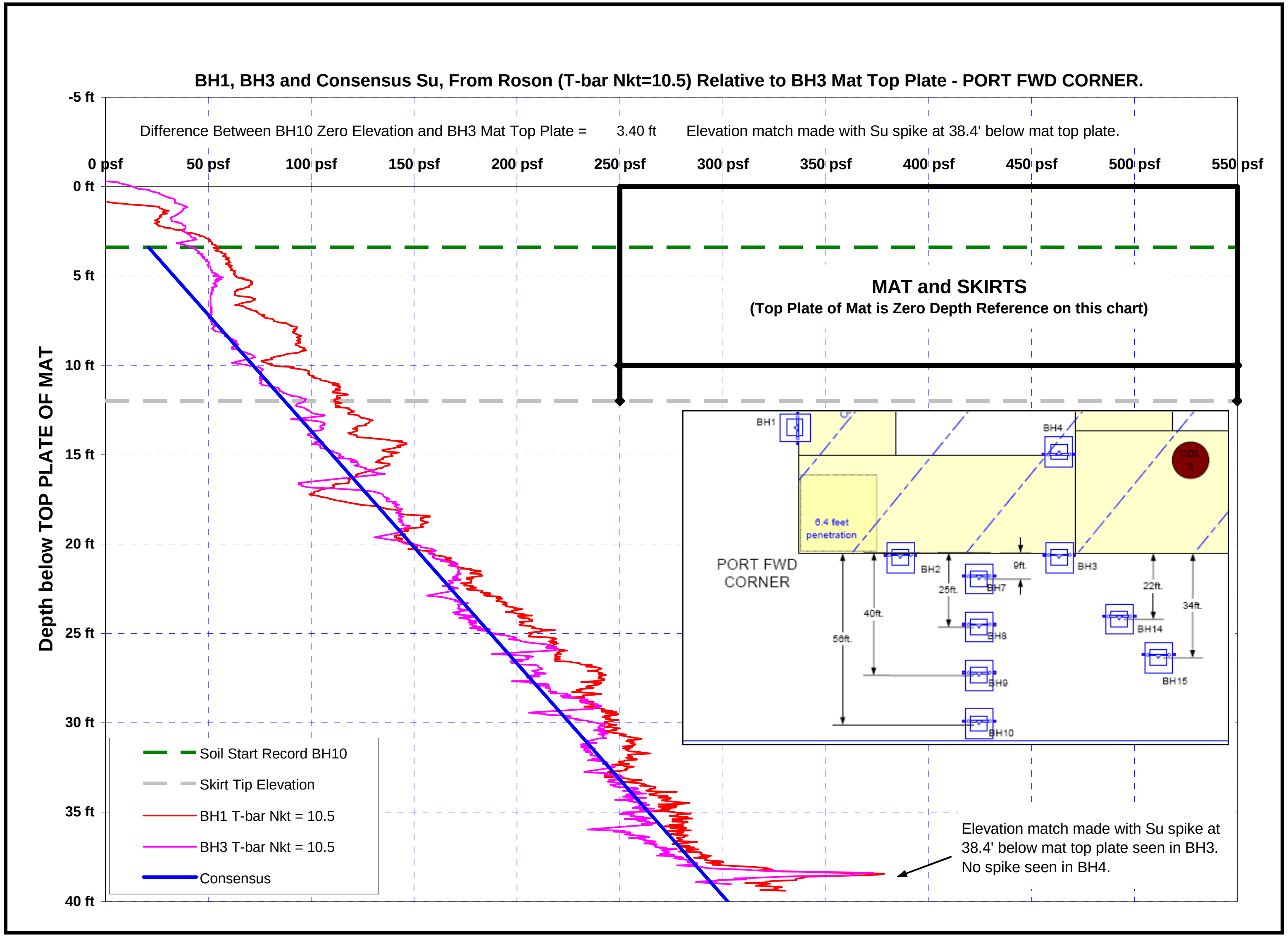

FIGURE 8 – Sample of Data From BH1 and BH2 on Port Forward Corner of Mat (T-Bar Results Only)

Soil at Mat After Installation

The borehole log data from the Roson was processed by Fugro and reviewed by a team of expert geotechnical engineers providing advice to the project. This team included Jean Audibert, Alan Young, Dan Spikola, Don Murff, Jack Templeton, Vladimir Rapoport and Steve Neubecker. The consensus opinion was that the soils in the mat effected zone have strengths are spatially uniform, and the soil has not been disturbed to any significant degree around the perimeter of the mat. The average strength profile selected for overturning stability analyses increases linearly with depth from 21 psf (1.0 kPa) at the seafloor to 324 psf (15.5 kPa) at 39.4 ft (12.0 m). This "new" strength profile increases linearly with depth at a rate of 7.72 psf/ft. This rate of strength increase is very similar to that considered to have existed before the structure was installed.

Figure 8 shows the "consensus" soil strength plotted on a strength versus depth chart with Fugro T-bar data plotter from the top plate elevation of the mat. Note that the soil heave around the mat sides and the interpreted original sea level are indicated in Figure 8.

Final Mat Penetration and Bearing Capacity Reconciliation

Table 3, below, shows the equilibrium depth at which the mat would find a bearing capacity of 600 psf (28.7 kPa) using the consensus soil strength profile (and neglecting overburden pressure, neglecting the heaved soil and neglecting the skirts that might make the depth for the foundation "bottom" two feet below the mat bottom plate) is 5.64 feet (1.72 m)

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.996	kappa.Baverage/cu.0	4.192	4.192
Nc.Kc	5.12	shape factor for pure vertical on kappa soil, scv	-0.052	-0.052
Depth Mat Bot.	5.64 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0035	-0.0035
Skirt Depth	0.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.048	0.048
Av. q_ult	600 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.996	0.996

TABLE 3 - Iterated Solution for Equilibrium Depth – Skirt Depth Ignored (Consensus Soil Strength)

From careful interpretation of the Fugro borehole logs, the actual mat bottom plate average penetration depth beneath the original undisturbed sea bed is a minimum of 6.5 feet (1.98 m) and is likely to be at least 1 foot (0.3 m) deeper than this. From the reference of the wellhead platform the estimated penetration is two feet deeper than this. It is noted that the elevation of the Roson machine used for pushing the rods into the sea bed had a precisely known elevation relative to the top surface of the mat plate. From this elevation a spike in the CPT and T-bar shear strength profiles enabled an accurate measurement of the depth of this "spike elevation" relative to the top of the mat. The depth of the spike elevation beneath the Roson locations away from the mat was determined at each borehole. However, the depth to which the Roson bottom frame penetrated the sea bed could not be accurately determined at any location away from the mat.

The crane operator attempted to lower the Roson, weighing around 5 tonnes in water, until he cold see his hook load reduce to about 1 tonne. This was not a very precise measurement and the 1 tonne target was initially provided to ensure that the 40 feet (12 m) high Roson frame remained upright during rod and instrument pushes at each location.

With the Consensus soil shear strength profile and the Roson footprint area of around 40 sqft (4.0 sqm) the penetration under a 4 tonne load, or around 220 psf, would have been significantly more than 1 foot (0.3 m) indicating that the mat was penetrated more than 7.5 feet. For the purpose of this paper an Average Roson Penetration into the sea bed of 1.5 feet is assumed at all locations not on the top plate of the mat. This results in an average mat penetration of 8.0 feet and is more in line with the 8.5 feet estimated from the comparative elevation of the wellhead platform (that itself has some doubt).

With a mat penetration of 8.0 feet (2.44m) the ISO calculation method indicates the soil would provide the mat with an average ultimate bearing pressure of 716 psf (34.3 kPa) without considering the buoyancy, or overburden term. Table 4 summarizes this calculation.

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.998	kappa.Baverage/cu.0	3.270	3.270
Nc.Kc	5.13	shape factor for pure vertical on kappa soil, scv	-0.035	-0.035
Depth Mat Bot.	8.00 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0024	-0.0024
Skirt Depth	0.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.067	0.067
Av. q_ult	716 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.998	0.998

TABLE 4 – Bearing Capacity Solution for Consensus Soil Shear Strength with Mat Penetration 8.0 feet, no overburden pressure.

If the bottom of the foundation is considered as being it the bottom of the skirts, the ultimate bearing capacity predicted by the ISO method (neglecting overburden pressure) is 812 psf (38.88 kPa). The calculation is summarized in Table 5, below.

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.998	kappa.Baverage/cu.0	2.756	3.270
Nc.Kc	5.13	shape factor for pure vertical on kappa soil, scv	-0.023	-0.035
Depth Mat Bot.	8.00 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0016	-0.0024
Skirt Depth	2.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.083	0.067
Av. q_ult	812 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.998	0.998

TABLE 5 – Bearing Capacity Solution for Consensus Soil Shear Strength with Mat Penetration 8.0 Feet and Considering Foundation Bottom to be at Skirt Bottom, no overburden pressure.

Table 6 shows the ISO ultimate bearing capacity if the skirts are neglected and the overburden effect is included. The predicted value for q_ult is 916 psf (43.86 kPa).

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.998	kappa.Baverage/cu.0	3.270	3.270
Nc.Kc	5.13	shape factor for pure vertical on kappa soil, scv	-0.035	-0.035
Depth Mat Bot.	8.00 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0024	-0.0024
Skirt Depth	0.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.067	0.067
Av. q_ult	916 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.998	0.998

TABLE 6 – Bearing Capacity Solution for Consensus Soil Shear Strength with Mat Penetration 8.0 feet, no skirt effect, with overburden pressure.

If the foundation bottom is considered to be at the bottom of the skirts and the overburden pressure (to the mat bottom plate level) is accounted for, the ISO method predicts a value for q_ult of 1012 psf (48.45 kPa) as shown in Table 7.

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.998	kappa.Baverage/cu.0	2.756	3.270
Nc.Kc	5.13	shape factor for pure vertical on kappa soil, scv	-0.023	-0.035
Depth Mat Bot.	8.00 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0016	-0.0024
Skirt Depth	2.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.083	0.067
Av. q_ult	1012 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.998	0.998

TABLE 7 – Bearing Capacity Solution for Consensus Soil Shear Strength with Mat Penetration 8.0 feet, full skirt effect, with overburden pressure.

Soil Strength Measurements at the Mat Bottom Plate Elevation.

T-bar results (using Nkt =10.5) for 4 boreholes with the Roson on the mat edge top plate are shown in Table 8. The locations of the boreholes are indicated in Figure 6. The average soil undrained shear strength at the level of the mat bottom plate is 81 psf (3.88 kPa). Taking this as the value to use in the ISO calculation for a soil having increasing strength with depth (the Davis and Booker solution) using the consensus soil strength kappa value, and neglecting both skirt effects and the overburden pressure effect, the ultimate bearing capacity of the soil beneath the mat, q_ult, is found to be 706 psf (33.8 kPa). The depth of the mat bottom plate would be 7.8 feet (2.38 m) beneath the original sea bed in this case, as is illustrated in Table 9.

SUMMARY OF SI Su RESULTS (from T-bar Nkt = 10.5) at Four Boreholes and Mat Center					
Borehole -->	BH1	BH3	BH4	BH6	Center average
Su, Bottom Plate observed	85.0 psf	70.0 psf	85.0 psf	83.0 psf	80.8 psf
Su, Skirt observed	115.0 psf	93.0 psf	113.0 psf	93.0 psf	103.5 psf
Su, Bottom Plate +9ft observed	145.0 psf	145.0 psf	140.0 psf	141.0 psf	142.8 psf
Su, Skirt +9ft observed	152.0 psf	160.0 psf	150.0 psf	150.0 psf	153.0 psf
Bearing Capacity at 6 x Su Botom Plate +9ft (no buoyancy)	870.0 psf	870.0 psf	840.0 psf	846.0 psf	856.5 psf
Bearing Capacity at 6 x Su Skirt +9ft (no buoyancy)	912.0 psf	960.0 psf	900.0 psf	900.0 psf	918.0 psf

TABLE 8 – Measured Soil Strengths from T-Bar Tests at Mat Edge.

Control Parameters		Coefficients to get Kc	scv, dc and K for bottom of skirt	dc and K for bottom of mat
Nc	5.14	Note that dc is calculated but set to zero in accordance with ISO guidance		
Kc	0.998	kappa.Baverage/cu.0	3.332	3.332
Nc.Kc	5.13	shape factor for pure vertical on kappa soil, scv	-0.037	-0.037
Depth Mat Bot.	7.80 ft	sc, shape coefficient = scv x 0.2*B/L	-0.0025	-0.0025
Skirt Depth	0.00 ft	dc, depth coefficient = 0.3*atan(depth/width)	0.066	0.066
Av. q_ult	706 psf	Kc = 1 + sc + dc - ic (dc = ic = 0)	0.998	0.998

TABLE 9 – Value for q_ult using cu =81 psf at Mat Bottom Plate Level, ignoring skirt and overburden effects.

Rationalization of Mat Penetration and Bearing Capacities

So how can these ultimate bearing capacities be reconciled with the known maximum average pressure achieved during preload of just 600 psf (28.7 kPa)?

Some possible explanations put forward by the geotechnical experts involved with this project, including:

Don Murff, Consultant to GEMS

Alan Young, GEMS

Dan Spikula, GEMS

Jean Audibert, Quest Geo-Technics

Jack Templeton, Sage USA

Vladimir Rapoport, Consultant

are listed below:

- The rocking of the mat during installation probably induced significantly higher bearing pressures than the maximum average value.
- Some consolidation has taken place since installation (about 9 months).
- The skirt effects are difficult to quantify.
- The overburden pressure effect is uncertain.

Of these possible explanations the rocking seems likely to have been the most effective in apparently getting the mat down into deeper stronger soil than would have probably been the case if the mat had been placed without rocking.

It must also be added that the bearing capacity for an infinitely long strip foundation would not be expected to be the same as that for a foundation with the same area but shaped like the Maleo mat. However, the mat shape would possibly give a higher capacity for the same area and therefore penetrate less.

Additionally it should be added that the interpreted in-situ CPT, T-bar and shear vane tests could all be over-estimating the strength of the soft clay at the site.

Summary of Bearing Capacity Results

Table 10, below, compares the calculated bearing pressures with the known maximum average applied bearing pressure of 600 psf (29 kPa). It seems likely that an engineer might conclude from the results of the 2007 detailed local SI in the mat effected zone that the ultimate bearing capacity that the mat now (at the time of the SI, May, 2007) offers is between 50% and 66% larger than that was proven during pre-loading.

COMPARISON OF BEARING CAPACITIES				
Condition		q_ult		q_ult Ratio
Average maximum bearing pressure at pre-load		600 psf	29 kPa	Base
ISO bearing pressure using ave. cu at mat bottom plate		706 psf	34 kPa	18%
	As above, with overburden pressure	901 psf	43 kPa	50%
As above, with overburden pressure + full skirt effect		997 psf	48 kPa	66%

TABLE 10 – Comparison Summary of Mat Bearing Capacities.

Calculation of Mat Overturning Resistance – Failure Mechanism

On soft clay, as at the Maleo site, a mat supported rig may be most prone to overturning (toppling) by either a deep-seated slip circle failure mechanism or by a progressive bearing capacity failure beginning with local failure around the mat edges. This latter type of failure, essentially a Prandtl strip footing failure was considered to be the first failure mode for overturning for the Maleo MOPU. However, the ultimate capacities of the mat to resist both failure modes were investigated. This paper does not attempt do give guidance as to when a mat with cutouts should be treated as a series of strips, or when the cutouts are small enough that it should be treated as a single equivalent slab without cutouts.

The strip footing failure mechanism on a normally consolidated marine clay that increases in strength with depth may be self-limiting in that, as with initial penetration when going onto location. In the vertical penetration case, the soil may be yielded as indicated in Figure 7, and the sea bed raised up at the sides of the mat "strips" as was found at the Maleo site. Various arguments can be made that the soil will or will not follow the displacement routes indicated by the arrows in Figure 7. Flow of the softest surface soil from beneath the mat and around the skirts is difficult to predict. It seems likely that some soft soil will be trapped under the mat bottom plate in the region between the skirts seen in Figure 5.

As the local failure beneath the mat strips causes the strips to penetrate deeper, the resistance to further penetration increases. However, so does the overturning moment as a consequence of the lateral shifting center of gravity high above the mat base. Conversely, if a deep seated slip circle failure is initiated, the collapse(toppling) of the structure may be rapid and may not involve any increased resistance once it commences.

An elegant argument for an idealization of the structure and foundation as a rigid- plastic system and to application of the upper bound method of plasticity to determine the overturning capacity under wave and wind loading was made by Don Murff, a consultant to the Maleo Project. The resistance to overturning calculated by Don's method is nearly identical to that which can be predicted by considering the ultimate bearing capacity beneath each strip in isolation and the maximum overturning moment such resistance can generate. This method is presented in this paper. Don's more elegant method will be presented in future papers.

Strip Foundation Method

The strip foundation method, as opposed to the slip circle method, considers the ultimate bearing capacity of each strip, represented in this paper as a simple single force acting vertically the center of each strip. The overturning resistance is taken as the sum of each force multiplied by its horizontal distance, or lever arm, from the assumed horizontal axis of rotation. The moment calculated from the platform weight (minus buoyancy) multiplied by its lever arm from the axis of rotation may add or subtract to the overturning moment caused by the environmental loading, generally taken as the combined horizontal wind, wave and current forces and their vertical distance (or lever arm) above the assumed horizontal axis of rotation.

The factor of safety against overturning, or OTSF, is defined as:

OTSF = (SRmoment – Wmoment)/OTmoment

Where:

SRmoment	=	soil ultimate capacity resisting moment
Wmoment	=	(weight – buoyancy) moment
OTmoment	=	overturning moment from environmental forces

It must be noted that the above definition of the OTSF is more accurately a definition of first yield in typical normally consolidated soft clays, where the increasing strength with depth may result in increasing resistance following first yield as deeper penetration occurs.

Figure 9 shows the section of strips for this paper for the case of lateral overturning. The axis of rotation may initially be considered as being along one edge and at the level of the base of the mat, although arguments exist that it might be lower than this, especially with mat effects. It may also be argued that the axis of least resistance to rotation may be above the mat base.

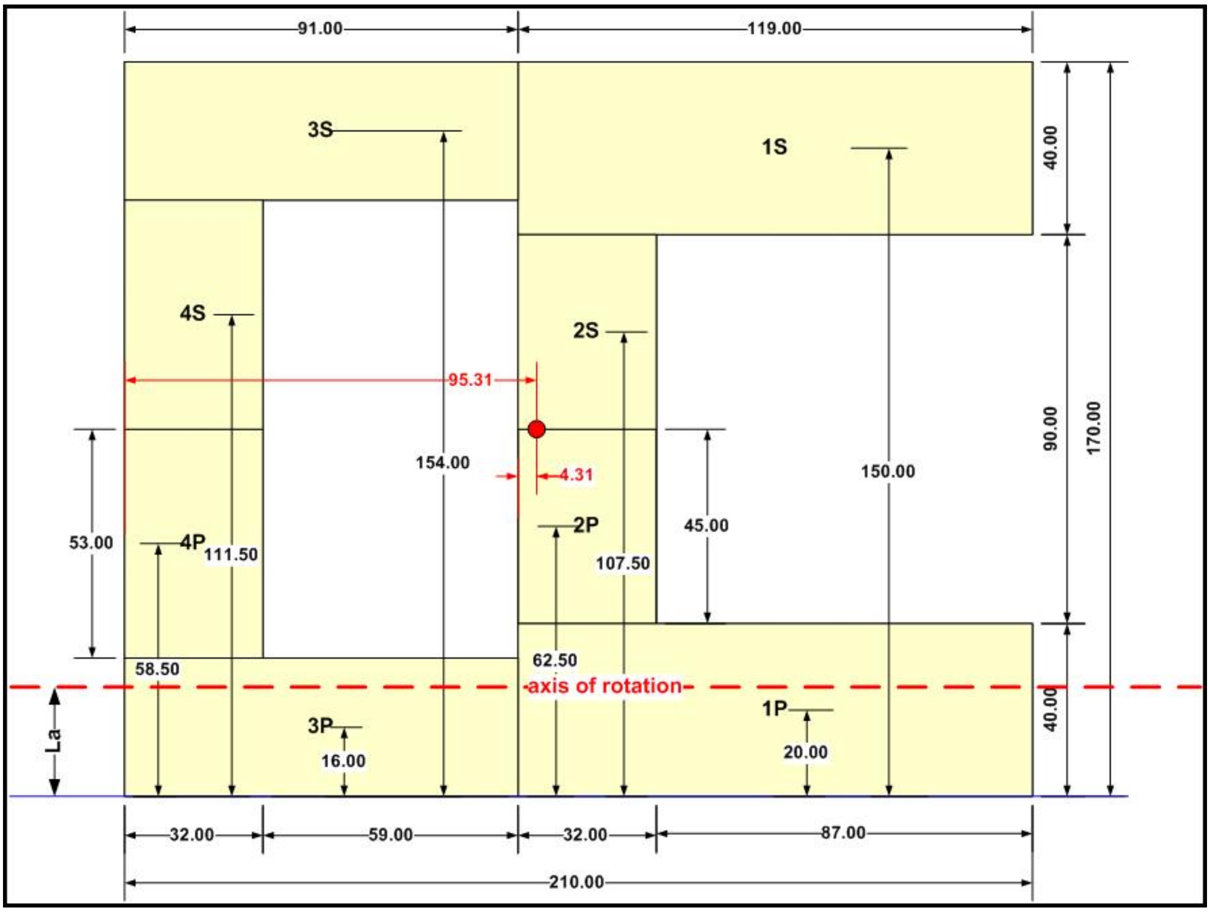

FIGURE 9 – Definitions of Strips and Axis of Rotation

The distance of the rotation axis from the edge of the mat, defined here as La, is found that results in the lowest value for the OTSF.

Suction, or resistance to uplift, may be considered as contributing to the OTSF, especially for individual waves, or may be omitted. The lever arm distance will be effected by uplift.

It is important to note that the strip footing method in this paper is not intended to deal with hard sea beds where the ABS MODU method of computing OTSF values considering toppling over a leeward edge may be appropriate.

Spreadsheet Implementation of OTSFs

Table 11, below, shows the individual strip conditions for analysis without overburden pressure and with a mat bottom plate penetration of 6.5 feet into the consensus soil shear strength profile, without skirt effects.

INVESTIGATE LATERAL OVERTURNING (ABOUT A LONGITUDINAL AXIS)									
Mat Area Name	Mat Areas	y-center	y-area-dist	Nc.Kc	Depth mat bot.	cu at mat bottom	Depth skirt bot.	cu at bottom of skirt	Overburden pressure force at bottom of mat
1S	4760 sqft	130.00 ft	618800 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
1P	4760 sqft	0.00 ft	0 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
2S	1440 sqft	87.50 ft	126000 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
2P	1440 sqft	42.50 ft	61200 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
3S	2912 sqft	134.00 ft	390208 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
3P	2912 sqft	-4.00 ft	-11648 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
4S	1696 sqft	91.50 ft	155184 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
4P	1696 sqft	38.50 ft	65296 ft^3	5.13	6.50 ft	71 psf	6.50 ft	71 psf	0 kip
Totals	21,616 sqft	65.00 ft	1405040 ft^3	5.13	6.50 ft				**0 kip**
	0.00	friction multiplier on skirts (0.0 = zero friction, 1.0 = reduced friction as below)					**Calculated Available Soil Force**		**13,891 kip**
	0.00	friction multiplier on mat sides (0.0 = zero friction, 1.0 = reduced friction as below)					**Force on bottom at max pre-load**		**12,970 kip**

TABLE 11a - Example Spreadsheet Computation of Ultimate Bearing Capacities of Mat Strips

Booker and Davis Calculations							overburden pressure force	Resisting Moments		Bearing force available
Mat Area Name	Effective widt	Cu-base	kappa.B/4	kappa.B/cu.0	F	q_ult		No Uplift	With Uplift	
1S	40.0 ft	71 psf	77 psf	4.33	1.51	666 psf	0 kip	412,392 kip-ft	412,392 kip-ft	3172 kip
1P	40.0 ft	71 psf	77 psf	4.33	1.51	666 psf	0 kip	00 kip-ft	00 kip-ft	3172 kip
2S	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	78,608 kip-ft	78,608 kip-ft	898 kip
2P	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	38,181 kip-ft	38,181 kip-ft	898 kip
3S	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	243,439 kip-ft	243,439 kip-ft	1817 kip
3P	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	00 kip-ft	7,267 kip-ft	1817 kip
4S	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	96,815 kip-ft	96,815 kip-ft	1058 kip
4P	32.0 ft	71 psf	62 psf	3.47	1.46	624 psf	0 kip	40,736 kip-ft	40,736 kip-ft	1058 kip
TOTALS							**0 kip**	**910,171 kip-ft**	**917,438 kip-ft**	**13,891 kip**

TABLE 11b – ISO (Davis and Booker) Calculation of Strip Bearing Capacities.

SUMMARY RESULTS FOR OVERTURNING ABOUT A LONGITUDINAL AXIS				No Uplift	With Uplift		
Overturning Resistance from Bearing Pressure Beneath Mat (Skirt Level)				910,171 kip-ft	917,438 kip-ft		
Add Contribution from Skirt External Cohesion				00 kip-ft	00 kip-ft		
Add Contribution from Mat Degraded External Cohesion				00 kip-ft	00 kip-ft		
Total Overturning Resisting Moment About Edge				**910,171 kip-ft**	**917,438 kip-ft**		
Overturning Axis Distance from Mat Edge			20.00 ft	<<--This number is calculated by clicking the macro buttom			
Environmental Moment (horizontal force)				163,774 kip-ft	163,774 kip-ft		
Weight moment	**9885 kip**	**65.00 ft**		642,524 kip-ft	642,524 kip-ft		
Total Overturning Moment About Edge				**806,298 kip-ft**	**806,298 kip-ft**	**Revised OTSF = (Ms-Mw)/Me**	
Calculated total vertical soil force available (side friction and bottom bearing)					**13,891 kip**	**no uplift**	**with uplift**
Eccentriciy	**16.57 ft**		Old Safety Factors	1.129	1.14	**1.63**	**1.68**

TABLE 11c – OTSFs for Lateral Overturning about Longitudinal Axis Parallel to Long Side of Mat.

MALEO PRODUCER ULTIMATE SOIL CAPACITIES AND OVERTURNING RESISTANCES

Author:	W.P. Stewart
Date Created:	6/15/06
File Name:	Overturning Simple Pressures Rev1b.xls
Last Modified:	6/6/07
Modified by:	WPS

Clicking the button below causes the worksheet to seek the axis location about which the overturning moment ratios are minimised for each of the three overturning outer mat edges considered.

The ratios in the cells below are calculated with the lowest values found after the button to the right is clicked.

Set Lever Arms 1, 2 & 3 to give min.moment ratios

Lever Arms 1, 2 & 3 w/uplift	Uplift Ratio w/(w/o)	Lever Arms 1, 2 & 3 w/o uplift	Rev. Ratio w/uplift	Revised Ratio w/o uplift	
20.00 ft	1.0272	20.00 ft	1.679	1.634	Lateral
43.50 ft	1.0000	43.50 ft	1.617	1.617	Longitudinal (Bow down)
16.00 ft	1.0745	61.50 ft	2.083	1.939	Longitudinal (Stern down)

SOIL PROPERTIES FOR DESIGN	
Gamma at mudline, (25)	0.00 lb/cuft
Gamma at 39ft depth, (34)	0.00 lb/cuft
Gamma_z = 25 + (34-25)/39 x z	
Su at surface	21.00 psf
Su at depth D_ref	324.00 psf

Soil Stength Graph data	Depth	Su
	0.00 ft	21.00 psf
Depth D-ref -->	39.40 ft	324.00 psf
kappa, Strength gradient		7.690 psf/ft

Skirt friction	00 kip
Mat Friction	00 kip
Bearing Force	13,891 kip
Total Capacity with skin friction	13,891 kip
Average Bearing pressure	643 psf
Overburden pressure	0 psf
Average Pressure for OTSF	643 psf

Control Parameters	
Nc	5.14
Kc	0.997
Nc.Kc	5.13
Depth Mat Bot.	6.50 ft
Skirt Depth	0.00 ft
Av. q_ult	643 psf

Move Rotation Axis 20.00 ft

TABLE 11d – Summary for OTSFs and Lever Arms for 6.5ft penetration of Mat Bottom Plate, No overburden, No Skirt Effects.

A comparison of bearing pressure ratios was shown in Table 10 for several conditions considered for the Maleo mat foundation. Table 12, below, shows a comparison of the OTFS ratios for the same conditions, taking the known 600 psf bearing pressure as the base case.

COMPARISON OF OTSFs			
Condition	**q_ult**	**OTSF**	**OTSF Ratio**
Average maximum bearing pressure at pre-load	600 psf	1.70	Base
ISO bearing pressure using ave. cu at mat bottom plate	706 psf	2.24	32%
As above, with overburden pressure	901 psf	3.23	90%
As above, with overburden pressure + full skirt effect	997 psf	3.72	119%

It is noted that OTSFs are much more sensitive to variations in the analysis assumptions than are the bearing pressures themselves.

Effect of Pre-Loading the Maleo Producer

The normal operating average bearing pressure exerted by the Maleo Producer on the sea bed is around 457 psf. The OTSF that would result from using this as the ultimate bearing capacity of the soil, i.e. if no pre-load had been undertaken and no detailed site investigation data was known after installation, is 0.97. The effect of increasing the bearing pressure to 600 psf at maximum pre-load is seen, in this case, to increase the OTSF to be greater than the minimum required value by ABS of 1.5.

THIS MANUSCRIPT IS NOT COMPLETE AND IS TO BE REVISED.

REFERENCES

1. DAVIS, E. H. and BOOKER, J. R. The Effect of Increasing Strength with Depth on the Bearing Capacity of Clays, Geotechnique, 23 (4), 1973, pp 551-563